Continuing Education in Engineering
The College of Engineering
University of California Extension

WATER RESOURCES ENGINEERING EDUCATIONAL SERIES

Program VII

POLLUTION OF COASTAL AND ESTUARINE WATERS

San Francisco, California
January 28-30, 1970

TABLE OF CONTENTS

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Section 9A

CONCEPTUAL DESIGN OF SUBMARINE OUTFALLS – II

HYDRAULIC DESIGN OF DIFFUSERS

by

Norman H. Brooks*

For a given ocean outfall, improvement of dispersal of sewage effluent is accomplished by use of a multiple jet manifold or diffuser at the end of the outfall sewer. If the sewage is discharged at a single port or "en masse", its dispersion and dilution will be slower than if it is discharged over a large area through a number of ports, as explained in the preceding lecture. In fact, without the use of multiple-outlet diffusers, other conditions being equal, much longer outfalls into deeper water are necessary to provide the same degree of dispersion and consequent shore protection.

An effective and simple type of diffuser is one which distributes the outflow through many ports over a large area with minimum head loss. The following discussion presumes a diffuser consisting of one long pipe, or several branching ones, with discharge ports at intervals along the pipes.

I. Basic Hydraulic Requirements

A. Flow Distribution. The division of the outflow between the various ports should be fairly uniform. If the diffuser is laid on a sloping sea bottom, it will be impossible to achieve uniform distribution between ports for all rates of flow. In such cases, it is advisable to make the distribution fairly uniform at low or medium flow, and let

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the deeper ports discharge more than the average port discharge during high rates of flow. To allow substantially less than average discharge from deeper ports is considered unsafe from the point of view of possible clogging of the deeper part of the diffuser.

B. Velocity in Diffuser. The flow velocity in all parts of the diffuser should be high enough to prevent disposition of any residual particles (remaining after primary sedimentation). For settled sewage, velocities of 2 fps to 3 fps at peak flow are adequate (but borderline) since these will tend to scour any material settled during low flow periods. If deposition takes place in any part of the diffuser over an extended period of time, the cross section of the pipe or outlet may become so constricted that locally the velocity will be reduced, a cycle that would accelerate the deposition process. The final result may be complete clogging of the terminal ports and failure of the diffuser to completely perform its dispersal function.

C. Ease in Cleaning. Even carefully designed diffusers will require occasional cleaning to remove any accumulated grease, slimes, and grit at intervals of two to five years. These accumulations tend to increase the apparent friction factor (mainly by decreasing cross-sectional area) thereby reducing flow from offshore ports and increasing flow from inshore port. Cleaning can be accomplished by flushing or pulling a ball through the line.

D. Prevention of Sea Water Intrusion. All ports should flow full in order to prevent the intrusion of sea water into the pipe. Sea water entering the pipe will be stagnant and will tend to trap grit and other settleable matter. Such deposits reduce the hydraulic capacity of the diffuser, thereby limiting its usefulness for future years when higher flows might be expected.

E. Total Head Loss. If effluent pumping is necessary or the available gravity head is limited, the total head loss in any proposed diffuser should be kept reasonably small. Additional head losses of a few feet are usually adequate.

F. <u>Port Design.</u> The outlet ports may quite satisfactorily be circular holes in the side of the pipe without nozzles or tubes or other projecting fittings. For optimum dilution the jets should discharge horizontally, with no initial upward component of velocity. The inside of the hole should preferably be bellmouthed to minimize clogging and to provide a discharge coefficient which will remain constant over a period of years.

II. Hydraulic Analysis

The hydraulic analysis of a multi-port diffuser is basically a problem in manifold flow. The following analysis illustrates how a diffuser can be designed, and demonstrates some basic principles peculiar to the design of ocean outfall diffusers.

A. Gravity Effects. Several gravity effects are important in diffuser flow. According to Rouse (44), for a circular orifice in a large tank, the Froude number should be greater than 0.59 in order for the orifice to flow full. For a rounded port, it is reasonable to take $F > 1$ as the criterion for flowing full. With every port in the diffuser flowing full, there is no way in which the sea water may re-enter the pipe, once initially expelled, and the diffuser will continue to remain full of sewage effluent. (For definition of F, see equation (1) of preceding lecture, "Conceptual Design of Submarine Outfalls - I, Jet Diffusion".)

In making hydraulic calculations, the pertinent pressure at any point in the diffuser is the pressure differential between the fluid inside the diffuser and the sea water outside at the level of the port. Working in reverse order from the deepest or farthest point of a diffuser backward, the decrease in depth tends to increase the pressure differential, in spite of the fact that the pressure of both sewage and sea water decreases. Henceforth, the use of the terms "pressure" and "pressure head" will herein refer to the pressure differential. The change of this pressure head due to a change of elevation of Δz will be equal to $\frac{\Delta\rho}{\rho}\Delta z$.

B. Characteristics of Flow from a Single Port. The hydraulic analysis of a diffuser is essentially a step-wise process starting at the extreme outer end. The ports are assumed to be far enough apart so that the flow in the vicinity of any one port is independent of the rest of the diffuser flow. The discharge from each port is figured separately in turn, and added to the quantity of flow carried by the diffuser pipe downstream. Between consecutive ports, the effective pressure head is increased by the amount of the friction loss and the density head ($\frac{\Delta\rho}{\rho}\Delta z$). The key to the problem is the analysis of lateral discharge from a port in the side of a pipe.

The rate of discharge, q, from an orifice or port in the side of a pipe (Fig. 9) is expressed by:

$$q = C_D a \sqrt{2 g E} \tag{11}$$

where q = side port discharge

C_D = discharge coefficient

a = port area (at smallest place) $= \pi d^2/4$

g = acceleration of gravity

$E = \frac{V^2}{2g} + \frac{\Delta p}{\gamma_f}$

Δp = pressure differential between inside and outside of pipe at location of port

γ_f = weight density of jet

V = mean velocity inside pipe

ν = kinematic viscosity

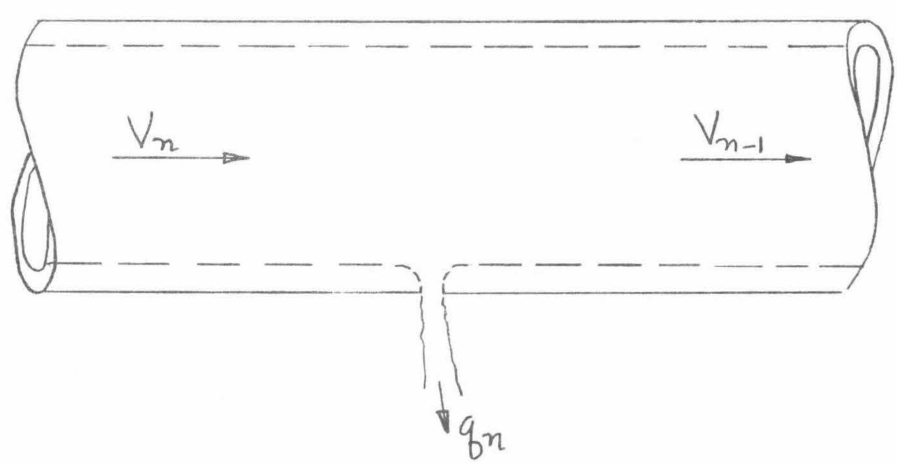


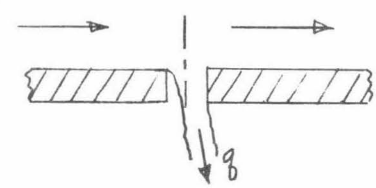
Fig. 9. Lateral discharge from port in the wall of a pipe.

In the neighborhood of the discharge port, it is assumed that there is no energy loss for the main flow in passing the port. In other words, there is perfect pressure recovery compensating for reduction in velocity head in the main flow because of the lateral discharge. McNown (42) has shown this to be a good assumption.

The discharge coefficient, C_D, is not a constant along the diffuser, but decreases as the velocity head ($V^2/2g$) becomes a larger part of the total energy (E).

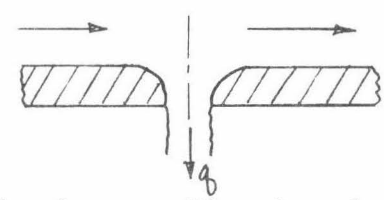
By laboratory experiments (under author's supervision) for Reynolds number $\frac{Vd}{\nu} > 20,000$, it has been found that for:

1) Sharp-edged ports, flowing full



$$C_D = 0.63 - 0.58\,\frac{V^2/2g}{E} \qquad (12)$$

2) For smooth bellmouth ports (with nozzle area contraction ≈ 4:1 or more) flowing full:



$$C_D = 0.975\left(1 - \frac{V^2/2g}{E}\right)^{3/8} \qquad (13)$$

These values supercede those given in Fig. 10 of reference 28 and apply only to small ports (< 1/10 of pipe diameter). The experimental result for a sharp-edged port (equation (12)) is within a few percent of what is obtained by extension of the theoretical analysis of branching flow by McNown and Hsu (43).

For small q, the velocities upstream and downstream of the port are approximately equal (that is, $V_n \approx V_{n-1}$) and either V_n or V_{n-1} may be used to calculate the ratio $\frac{V^2}{2g}/E$ for use in equations (12) or (13), although V_{n-1} is the more convenient.

C. <u>Calculation Procedure.</u> The calculation procedure used in the design of a diffuser may be formulated mathematically as follows:

Let: D = diameter of pipe;

d_n = diameter of nth port, counting n from offshore end;

a_n = area of nth port;

V_n = mean pipe velocity between nth port and $(n+1)^{th}$ port (see Fig. 9);

$\Delta V_n = V_n - V_{n-1}$ = increment of velocity due to discharge from nth port (or group of ports);

$h_n = \Delta p_n / \gamma$ = difference in pressure head between the inside and the outside of the diffuser pipe just upstream of nth port (expressed in feet of sewage);

$E_n = h_n + \frac{V_n^2}{2g}$ = total head at nth port (same either side by assumption above);

C_D = discharge coefficient for ports

q_n = discharge from the nth port;

h_{fn} = head loss due to friction between (n + 1) and nth port;

L_n = distance between (n + 1) and nth port;

f = Darcy friction factor;

Δz_n = change in elevation between (n + 1) and nth port (measured to center of port; positive when (n + 1) port is not as deep as the nth port);

ρ = density of sewage;

$\Delta \rho$ = difference in density between sea water and sewage

First it is necessary to select E_1; then q_1 for the first port is:

$$q_1 = C_D a_1 \sqrt{2 g E_1} = C_D \frac{\pi}{4} d_1^2 \sqrt{2 g E_1} \tag{14}$$

Next, one finds the pipe velocity

$$V_1 = \Delta V_1 = \frac{q_1}{\frac{\pi}{4} D^2} \tag{15}$$

and velocity head $V_1^2/2g$.

Proceeding to port no. 2, one finds E_2 by

$$E_2 = E_1 + h_{f1} + \frac{\Delta s}{s} \Delta z_1 \tag{16}$$

The ratio $\frac{V_1^2}{2g} / E_2$ is calculated for use in equation (12) or (13) to find C_D. Then

$$q_2 = C_D a_2 \sqrt{2 g E_2} \tag{17}$$

and

$$V_2 = V_1 + \Delta V_2 = V_1 + \frac{q_2}{\frac{\pi}{4} D^2} \tag{18}$$

This procedure is continued step by step back up the diffuser using the general relations:

$$q_n = C_D a_n \sqrt{2 g E_n} \tag{19}$$

$$C_D = \varphi \left(\frac{V_{n-1}^2}{2g} \Big/ E_n \right) \qquad \text{(12) or (13)}$$

$$\Delta V_n = \frac{q_n}{\frac{\pi}{4} D^2} \tag{20}$$

$$V_n = V_{n-1} + \Delta V_n \tag{21}$$

$$E_{n+1} = E_n + h_{fn} + \frac{\Delta s}{s} \Delta z_n \tag{22}$$

and

$$h_{fn} = f \frac{L_n}{D} \frac{V_n^2}{2g} \tag{23}$$

The procedure is readily carried out with a digital computer and a number of trial designs can be easily investigated.

If the port discharges and pipe velocities change slowly, it is expedient to make the stepwise calculations for small groups of ports. In this case, eq. (20) is changed to read

$$\Delta V_n = m \frac{q_n}{\frac{\pi}{4} D^2} \tag{24}$$

wherein m is the number of ports considered in a group.

By the nature of the calculations, it is apparent that one cannot decide on a particular total flow before starting the calculations. It is necessary to estimate the flow from the end port (q_1) which will correspond to the desired total flow.

D. Selection of Port Sizes and Pipe Sizes. During the process of the calculation, the designer is at liberty to change the pipe size, the port size, and/or the port spacing. To keep the velocity high enough at the end of the diffuser, it is sometimes necessary to reduce the size of the pipe in one or more steps from beginning to end of the diffuser. The size of the discharge ports may be varied in order to keep the discharge uniform from port to port. The spacing between ports is rather inflexible, inasmuch as practical considerations dictate that the spacing be either equivalent to the length of a pipe section or multiple or simple fraction thereof. The entire design process inevitably requires some trial and error arrangements in order to get one arrangement which is satisfactory at various total rates of flow.

For a diffuser which is laid at zero slope, the relative distribution of flow would be the same at all rates of discharge. This is because all the head terms are proportional to the square of the velocity. In that case, where there are no differential elevations, one calculation would suffice for all rates of flow. For example, to double the rate of flow, one would need only to quadruple all the heads and double all the velocities and discharges.

It is essential that the end of the diffuser pipes be bulkheaded, otherwise the flow will not be forced out of the discharge ports near the end of the diffuser, and an excess of flow will be discharged through the open-ended pipe. The bulkheads should be removable for flushing the line.

In the process of making the hydraulic calculations it was found that a good rule of thumb was to assure that the sum of all the port areas is less than the cross-sectional area of the outfall pipe. It is impossible to make a diffuser flow full if the aggregate jet area exceeds the pipe cross-section area, since that would mean that the average velocity of discharge would have to be <u>less</u> than the velocity of flow in the pipe. Experience with several diffuser designs indicate that the best area ratio (Σports: pipe) is usually about 1/2 to 2/3; these values are small enough to get good flow distribution among the ports, but not so small as to increase the total head unduly.

E. <u>Examples.</u> Examples of hydraulic designs of diffusers are given in references 28, 37 and 40.

F. <u>Summary.</u> A diffuser can be designed by calculating the port discharges one at a time starting with the offshore end. For discharge through lateral ports in a pipe, the discharge coefficient is a variable function of the ratio of the velocity head to the total head in the main pipe, as shown by equations (12) and (13). Balanced distribution of discharge among the ports can be secured by varying the port diameter. A necessary requirement in selecting port sizes is to keep the sum of port areas less than the cross-sectional area of the outfall, preferably only 1/2 to 2/3 as much.

III. Diffuser Loading

For comparison of diffuser pipe length of various outfalls, it is useful to compute the ratio of length in feet to design value of average daily dry-weather discharge in mgd. Some values of diffuser loading in recent designs are as follows (with year operation started):

	Ft. of diffuser/average dry-weather flow
County Sanitation Districts of Los Angeles County (at Whites Point)	
90" outfall (1956)	2400'/150 mgd = 16'/mgd
120" outfall (1965)	4440'/220 mgd = 20'/mgd
City of Los Angeles at Hyperion	
144" outfall (1960)	8000'/420 mgd = 19'/mgd
Metrópolitan Seattle at West Point	
96" outfall (1965)	600'/125 mgd = 4.8'/mgd
County Sanitation Districts of Orange County	
120" outfall (1970)	6000'/290 mgd = 21'/mgd

Acknowledgments

These notes have been prepared for the Water Resources Engineering Educational Series, Program VII, "Pollution of Coastal and Estuarine Waters," University Extension and the College of Engineering, University of California, Berkeley, Jan. 28-30, 1970.

These notes are essentially a condensation and revision of a portion of Rawn, Bowerman and Brooks (28).

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The following references from the preceding lecture were cited herein:

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44. Rouse, H., "Elementary Mechanics of Fluids," John Wiley and Sons, Inc., 1946, p. 105.